Liquefaction Induced Settlement of Structures

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ABSTRACT

Soil liquefaction following earthquakes leads to excessive damage to a wide variety of structures. Settlement and rotation of structures following liquefaction have been witnessed in many of the recent earthquakes. Investigation of the mechanisms of failure of structure when the foundation soil suffers either partial or full liquefaction is therefore very important. Dynamic centrifuge tests were conducted at Cambridge and elsewhere on different boundary value problems in which liquefaction of soil models was investigated. Excess pore pressure data and the settlement data for the particular structure that is being investigated are recorded during the centrifuge tests. In this paper the centrifuge test results from a range of structures will be considered. The co-seismic and post seismic settlement of structures will be considered separately along with the excess pore pressure recorded generated during the cyclic loading. It will be argued that the co-seismic component of the settlement is much larger than the post-seismic settlement in many of the structures considered. Accordingly a hypothesis that the hydraulic conductivity k of the liquefied soil during the earthquake shaking is much higher than the normal hydraulic conductivity is proposed. A discussion on the micro-mechanical reasons for this increased hydraulic conductivity is presented.

INTRODUCTION

Liquefaction of loose sandy and silty soils results in excessive damage to a wide variety of structures following major earthquakes. Many of the recent earthquakes such as the Kobe earthquake of 1995, the 921 Ji-Ji earthquake in Taiwan and the Turkey and Bhuj earthquakes of 2001 have all provided many examples of liquefaction induced damage. In Fig.1 the rotation of the Harbour Masters Tower (HMT) at the Kandla Port in India is shown. This building was supported on pile foundations and suffered a rotation of nearly 10^o following liquefaction and lateral spreading of the foundation soil. This case history was described in detail by Madabhushi et al (2009).

Fig. 1. Rotation of the HMT building at Kandla Port following the Bhuj earthquake

This type of examples are numerous but the underlying problem is in the understanding of the failure mechanisms of the structures founded on soils that either suffer full or partial liquefaction when subjected to earthquake shaking. To this end a number of researchers have investigated a range of structures that are founded on such soils. Dynamic centrifuge modeling is commonly used to study such problems on soil liquefaction and is particularly well-suited to determine the failure mechanisms. Equally full coupled finite element codes with soil models that can simulate excess pore pressure generation and soil liquefaction may be used to analyze boundary value problems, for example, SWANDYNE (Chan, 1988).

In this paper boundary value problems that involve different types of structures will be considered. In each case the results from a series of dynamic centrifuge tests will be presented. The principles of centrifuge modeling are now well-known and the scaling laws that relate the model behaviour with prototype structures were originally described by Schofield (1980, 81) and more recently by Madabhushi (2004). From the centrifuge test results considered in this paper, the excess pore pressure generation following earthquake loading and the settlement of structures will be highlighted. The main emphasis of the paper will be to delineate the co-seismic component of the settlement from the post-seismic settlement in each case. Based on these observations a hypothesis on the changes in the hydraulic conductivity of the liquefied soils will be proposed.

SINGLE DEGREE FREEDOM STRUCTURE

A series of dynamic centrifuge tests on a simple, single degree of freedom structure have been conducted by Mitrani and Madabhushi (2008, 2009). The model structure exerted a bearing pressure of about 55 kPa in 50g (50 × earth's gravity) centrifuge test. In this series of tests, the foundation soil below the structure was loose, saturated Hostun S28 sand. A benchmark test (BM1) was conducted on a centrifuge model, the cross-section of which is shown in Fig. 2.

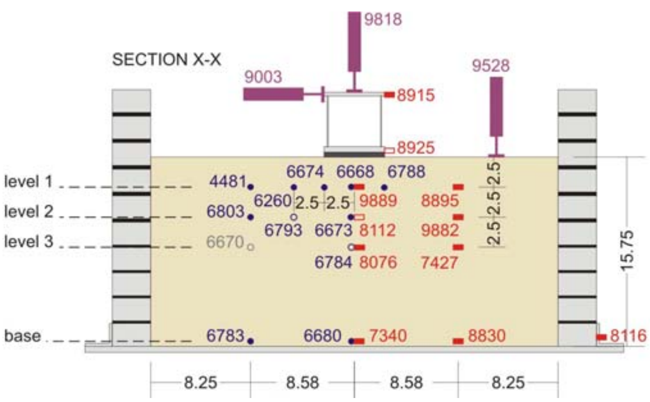

Fig. 2. Cross-section of the centrifuge model BM1

In subsequent centrifuge tests, the foundation soil was varied. Two particular tests will be considered here. The first variation was to consider a cemented zone below the SDOF structure that extends partially into liquefiable, loose sand deposit. The cross-section of this centrifuge model CZ1P is shown in Fig. 3. The second variation was to extend the cemented zone to a much deeper level into the liquefiable layer as indicated by the cross-section of model CZ1F in Fig. 4. All the centrifuge models are heavily instrumented. In Figs. 2 to 4 the location of accelerometers (indicted by rectangles) and miniature pore pressure transducers (indicated by solid circles) and LVDT's are shown. The dimensions are given at prototype scale. All the results presented in this paper will also be at prototype scale.

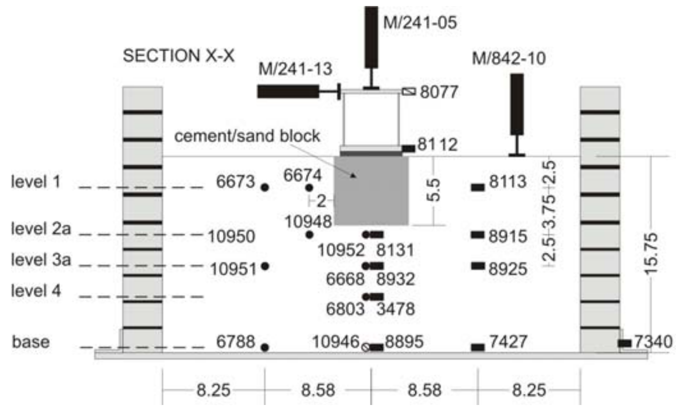

Fig. 3. Cross-section of the centrifuge model CZ1P

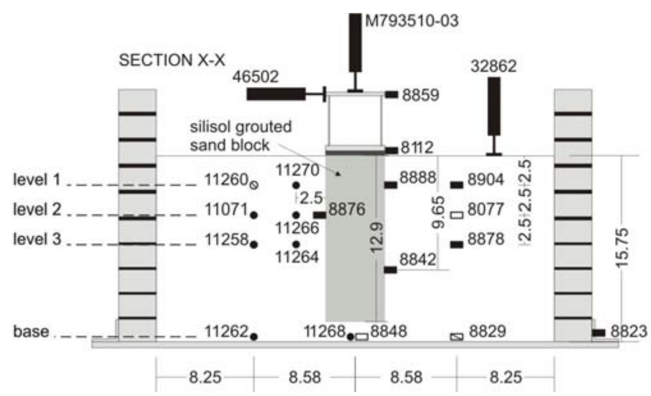

Fig. 4. Cross-section of the centrifuge model CZ1F

Settlement of the Structure

The most important parameter in liquefaction studies is the settlement suffered by the structure following the earthquake loading. In Fig. 5 the co-seismic settlement of the structure during the earthquake 3 (up to 25 sec) and post-seismic settlement immediately after the end of the earthquake loading are presented for all the centrifuge models described above. The results from the benchmark test BM1 are compared separately with the partial cemented zone tests (CZ1P) and the full cemented zone test (CZ1F). The settlement of the structure in CZ1F is smaller than in CZ1P and BM1 as would be expected. In each case it can be seen that the magnitude of the co-seismic settlements are quite large compared to the post-seismic settlement firstly for the free-field soil and secondly for the structure. It may also be noticed in Fig. 5 that the rate of settlement of the structure is also much larger in the co-seismic period compared to post-seismic period. Further, the magnitude and the rate of settlement of the free field soil surface is smaller compared to those of the structure in all the centrifuge tests BM1, CZ1P and CZ1F.

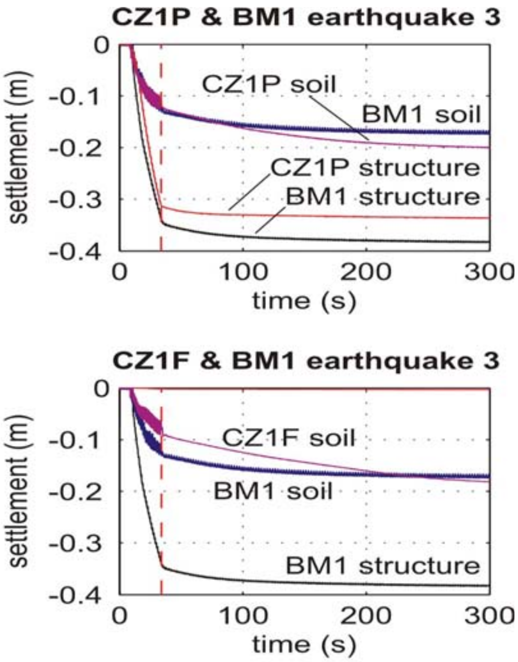

Fig. 5. Settlement of structure following earthquake loading

In Fig. 6 a comparison of the co-seismic and post-seismic settlements are presented for all the earthquakes fired (EQ1 to EQ3). In this figure the structural settlement is plotted as the difference between the cemented zone tests and the benchmark test BM1. In both the comparisons in Fig. 6 it can be seen that co-seismic settlement component (light blue) is much larger than the post-seismic component (light gray).

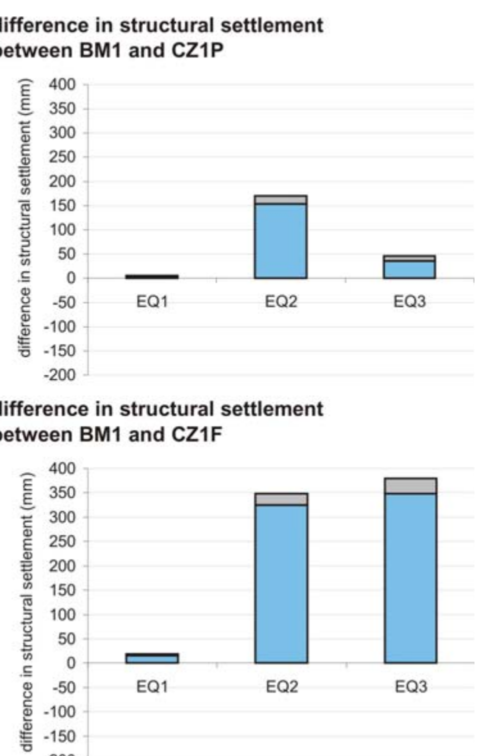

Fig. 6. Differences in settlement of structures in different centrifuge models

Excess Pore Pressures

The excess pore pressures generated within the soil models were recorded using PPTs. An example of the excess pore pressures recorded during earthquake 3 in the centrifuge model CZ1P is shown in Fig. 7. The location of the PPTs can be seen in Fig. 3 which lie at different levels below the ground surface indicated by levels 2 to 4. The pore pressure traces shown are for PPT's that are directly below the cemented zone. Clearly large excess pore pressures were recorded during this strong earthquake. The level of excess pore pressure required to fully liquefy the soil are approximately shown (as dashed lines) in Fig. 7. At level 2a the excess pore pressure trace shows some suction (drop in excess pore pressure) indicating a monotonic dilation of the liquefied soil (indicted by a dashed ellipse) as the structure and the cemented block start to settle into the liquefied ground). This effect is also seen at level 3a albeit being less pronounced.

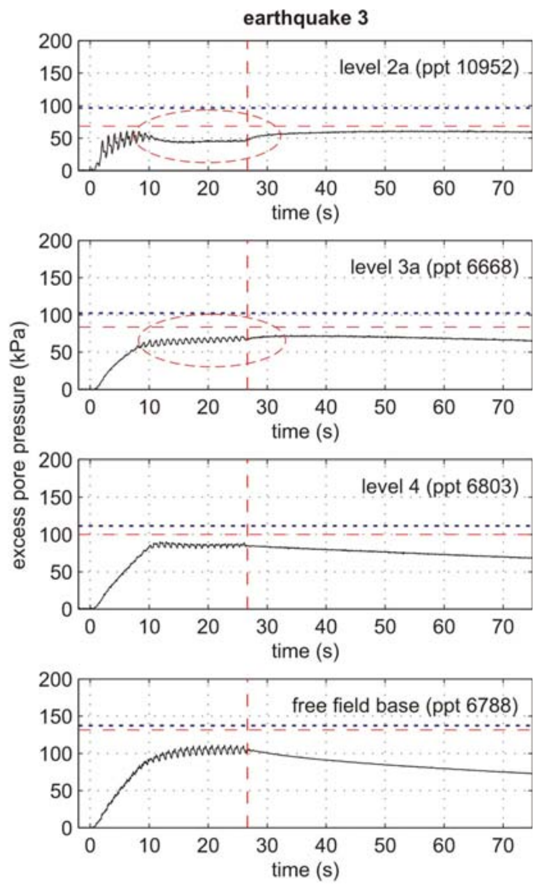

Fig. 7. Excess pore pressure records following earthquake loading in centrifuge model CZ1P

It is interesting to note that the excess pore pressures immediately after the end of the earthquake (t > 28 sec) are very similar to those during the later part of the earthquake (10 sec < t < 28 sec), once the excess pore pressures were generated in the initial period of the earthquake. This is in contrast to the co-seismic and post-seismic settlements described in previous section.

Structural Accelerations

The accelerations were monitored at different locations in the soil and on the structure as indicated in Figs. 2 to 4. In Fig. 8 an example of the acceleration-time histories recorded at the model base, at level 2 in soil and on the structure's base and top are presented for centrifuge model CZ1P for earthquake 3. In this figure it can be seen that the structural accelerations diminish significantly after the initial period of the earthquake (t > 10 sec). This ties up nicely with the generation of the excess pore pressures discussed earlier in that, once the excess pore pressures are fully generated and the soil below the cemented zone suffers liquefaction, the structure enjoys a relative isolation from the incoming shear waves.

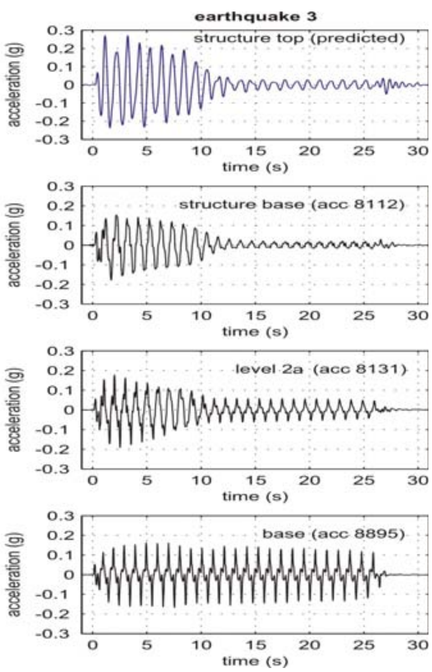

Fig. 8. Acceleration records in centrifuge model CZ1P

In Fig. 9 the acceleration records for the centrifuge model CZ1F are presented. In this case the cemented zone extends much deeper into the liquefiable sand layer. As a result the structure is not isolated from the incoming shear waves and is subjected to full shaking accelerations. This is confirmed by comparing the acceleration trace of base input to the acceleration recorded at the base of the structure of 0.2g. As a result the structure will vibrate vigorously and large accelerations of up to 0.4g are recorded at the top of the structure. This large amplification factor of '2' however depends on the natural frequency of the structure and the damping present in the model structure. The main point here is that the structural vibrations in this case are not isolated as the cemented zone conducts the earthquake shaking effectively, unlike the liquefied soil below the cemented zone in model CZ1P.

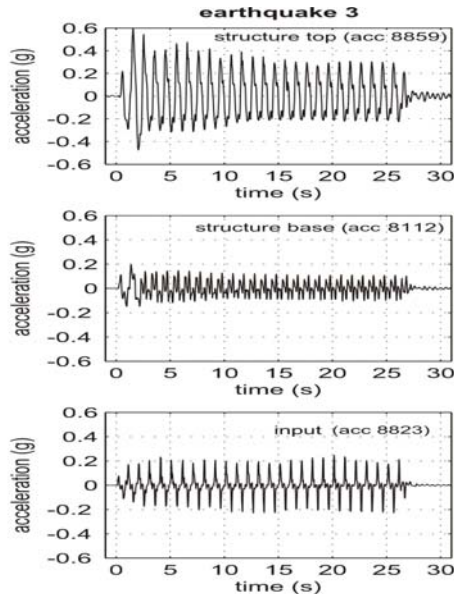

Fig. 9. Acceleration records in centrifuge model CZ1F

RIGID, HEAVY STRUCTURE

The next example that will be considered is that of a rigid, heavy structure on layered soil layers. Ghosh and Madabhushi (2005, 2007), Ghosh et al (2005) describe a series of dynamic centrifuge tests on rigid structures such as Nuclear Reactor Buildings (NRBs) founded on layered soil strata and subjected to earthquake loading. The model building exerted a bearing pressure of about 150 kPa on the foundation soil when tested at 50g's.

As in the previous centrifuge tests, the centrifuge models are heavily instrumented. A typical cross-section of a centrifuge model in this test series is shown in Fig. 10 along with the location of the instrumentation. All dimensions are shown in prototype scale. In this test a loose sand layer with relative density of 45% is sandwiched between dense sand layers with a relative density of 85%. In this series tests some of the centrifuge models had vertical stratification with the dense sand layer below the NRB model structure extended to the base of the model with loose, sand layers on either side.

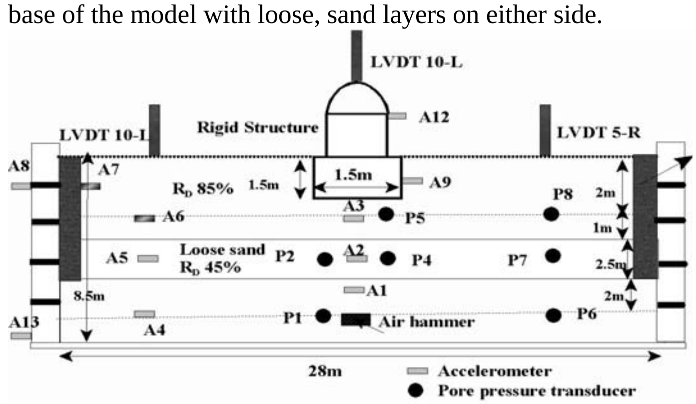

Fig. 10. Cross-section of a centrifuge model a rigid structure on stratified sand layers

Typical views of the centrifuge model of an NRB model structure on a homogeneous, loose sand layer, before and after the centrifuge test are presented in Fig. 11. Comparing the two views it can be seen that the model structure has suffered both rotation and settlement following the earthquake loading.

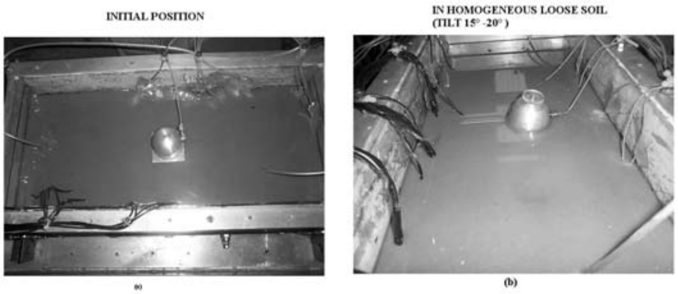

Fig. 11. Typical views of the centrifuge model before (left) and after (right) the centrifuge test

Settlement of the NRB Model Structure

The settlement of the model NRB structure is shown in Fig. 12 from different centrifuge models namely, homogeneous sand layer, horizontally stratified and vertically stratified soil layers. The co-seismic settlements are seen in the period between 10 sec to 40 sec. The post-seismic settlements are seen from 40 sec to 100 sec. As in the case of SDOF structure before, the magnitude of the co-seismic settlements of the NRB model structure is much larger than that of the post-seismic settlements. Similarly the rate of settlement is also much larger during the co-seismic period compared to the post-seismic period.

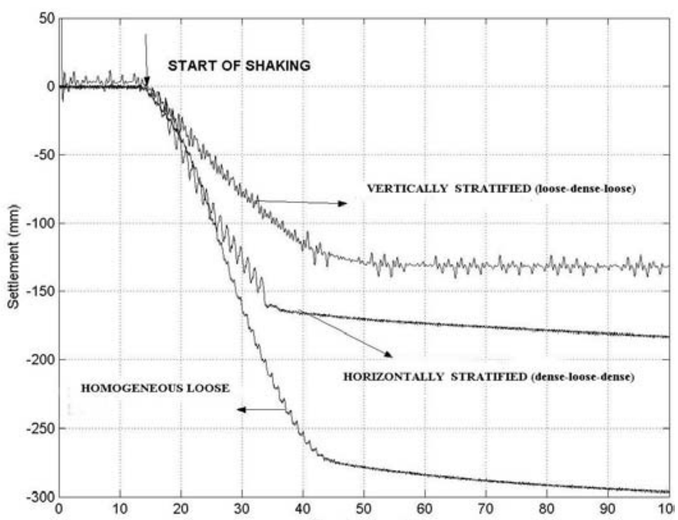

Fig. 12. Settlement of the NRB structure in different centrifuge tests

It is also interesting to note from Fig. 12 that the maximum settlement of the NRB model structure occurs when it is founded on homogenous soil layer. The settlement is smaller for horizontally stratified layer and this is further reduced in the case of vertically stratified soil layer, with dense zone below the NRB model structure. These results, while expected, confirm the veracity of the centrifuge test data.

Excess Pore Pressures

Excess pore pressures generated during earthquake loading are measured at different PPT locations shown in Fig. 10. A typical example of the excess pore pressure records is presented in Fig. 13 at selected locations. In the free-field the excess pore pressure is fully generated to match the initial effective stress and thereby ensuring full liquefaction as confirmed by trace P8 in Fig. 13. The excess pore pressure recorded by P5 just below the NRB model structure initially shows positive excess pore pressure generation. This is quickly suppressed by the monotonic dilation of the sand as the heavy structure starts to settle and subject the sand below it to monotonic shear stress (over and above the cyclic shear stresses generated by the earthquake loading). This is manifested as a reduction of the excess pore pressure. This monotonic dilation is much stronger than that observed earlier underneath the SDOF structure in Fig. 7, which is a much lighter structure compared to the NRB model structure. Following the end of the earthquake loading the rate of settlement of the structure is reduced as seen in Fig. 12. This reduces the monotonic shear stress and the excess pore pressure recorded by P5 starts to increase again due to migration of the pore fluid from free-field into this region below the structure.

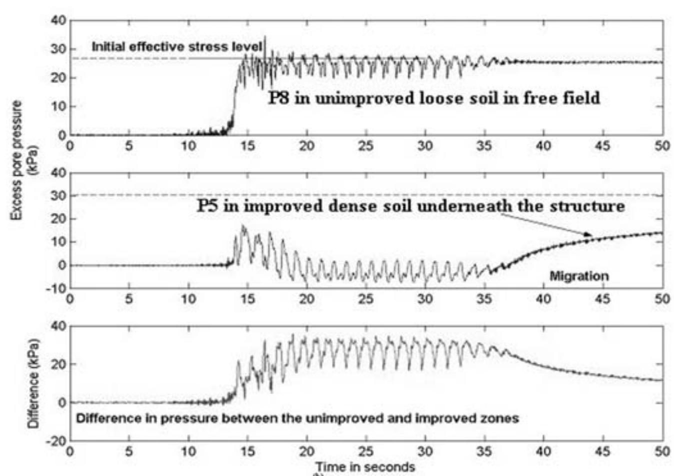

Fig. 13. Excess pore pressure records in different centrifuge model tests on the NRB model structure

In Fig. 13 the difference in excess pore pressure between the free-field and the region below the NRB model structure is plotted. This trace reflects the available pressure gradient that will set up and sustain the migration of pore fluid from free-field into the region below the structure, until pore pressure equalization occurs. Similar behaviour of pore fluid migration from loose sand (free-field) into dense sand (below structure) was also observed for vertically stratified soil strata, although these are not discussed here.

Structural Accelerations

The structural accelerations recorded at the base of the NRB model structure by accelerometer A9 in different centrifuge model tests are presented in Fig. 14. In this figure it can be seen that for the case of this structure founded on homogenous, loose sand deposit, the structural acceleration attenuates after the first few cycles from about 0.22g to 0.1g. However some coupling between the structure and the soil still exists as the NRB model structure is heavy and is embedded into the soil. Similarly for the case this structure founded on horizontally stratified soil, the structural accelerations show some attenuation. However, for the case of vertically stratified soil layer the structural accelerations shown no attenuation at all. Again these results are as expected, but serve to confirm the veracity of the centrifuge test data.

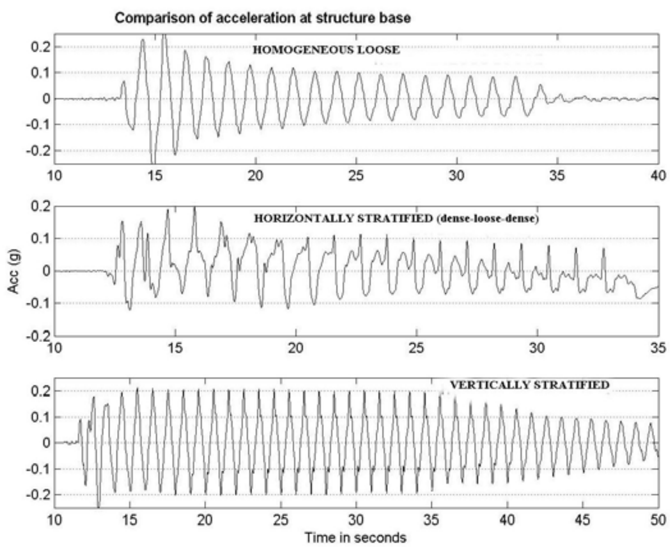

Fig. 14. Settlement of the NRB structure in different centrifuge tests

PILE FOUNDATIONS

The final example that will be considered in this paper is that of pile foundations in liquefiable soil layers. Haigh and Madabhushi (2005), Bhattacharya et al (2004, 2005) followed by Knappett and Madabhushi (2008, 2009a, 2009b) have investigated extensively the behaviour of pile foundations in liquefiable soils using dynamic centrifuge modeling. In this paper only the settlement aspects of the pile foundations will be emphasized. Therefore pile groups which pass through a liquefiable layer and are driven into an underlying dense sand layer are considered.

A schematic diagram of a typical centrifuge model of pile groups in layered soil strata is shown in Fig. 15. The axial load on the pile group is simulated by using a number of square blocks made out of brass. By changing their number, different amount of axial load could be modeled in each centrifuge test. In Fig.15 the pile cap for both the pile groups is clear of the ground surface. This allows for the free settlement of the pile groups. In other centrifuge tests the pile cap was made to rest on the ground surface. Clearly in those cases, additional bearing capacity from the pile cap is mobilized as the pile group tries to settle following soil liquefaction. The location of typical instrumentation used in these tests is also shown in Fig. 15. All the dimensions are shown at prototype scale.

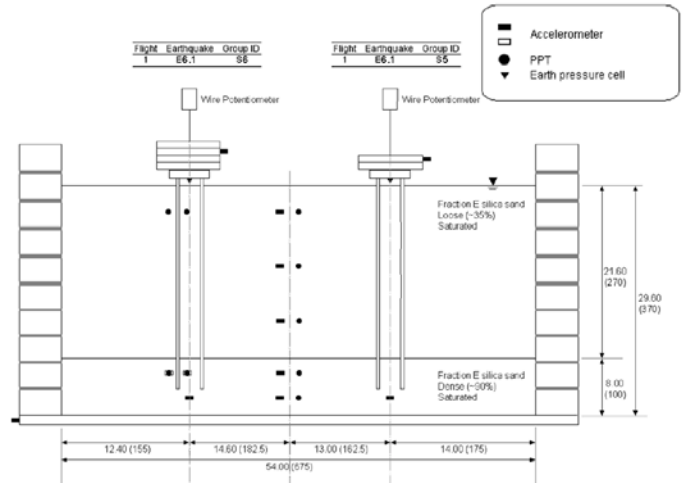

Fig. 15. Cross-section of a centrifuge model of pile groups

Excess Pore Pressures

An example of the excess pore pressures generated at different depths is presented in Fig. 16. As before, the horizontal dashed lines in this figure indicate the full liquefaction level i.e. when the excess pore pressure equals the initial total stress. As seen in Fig. 16, full liquefaction was achieved at all depths after the initial period of the earthquake (about t = 35 sec).

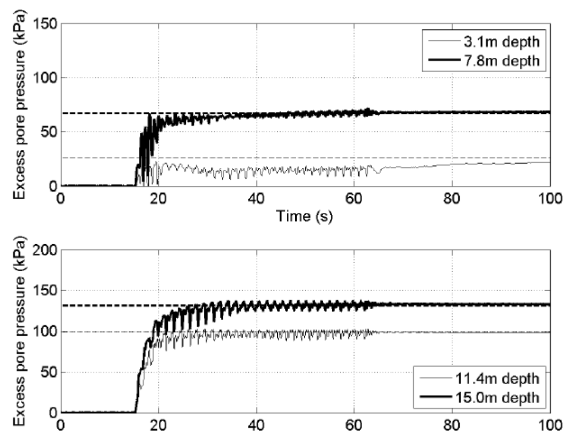

Fig. 16. Excess pore pressure generation in centrifuge models of pile groups

Settlement of the pile groups

As in the previous boundary value problems, the main focus of this paper is on the liquefaction induced settlement. These are considered for two sets of pile groups S1 and S3. Pile group S1 had the pile cap well above the ground surface as shown in

Fig. 15. The second pile group S3 had the pile cap in contact with the ground surface (not shown in Fig. 15). The settlement-time histories are presented for these pile groups in Fig. 17. It must be pointed out that the dynamic variations during the earthquake loading have been filtered from the settlement curves.

As observed in the other boundary value problems, the co-seismic settlements between 18 sec and 60 sec are much larger for both pile groups S1 and S3 compared to post-seismic settlements. In the case of S3 the post-seismic settlements are negligible as the pile cap bearing capacity is fully mobilized by that stage and the pile group does not show any more settlement. In the case of pile group S1 there is some post-seismic settlement that occurs beyond 60 sec. The rate of settlement is also much larger for both the pile groups in the co-seismic period compared to the post-seismic period as seen in Fig. 17.

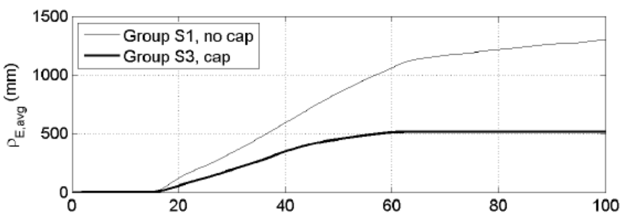

Fig. 17. Settlement of the pile groups following soil liquefaction

This trend of larger co-seismic settlement compared to the post-seismic settlements was observed for many other pile groups with different axial loads and soil strata thicknesses. Some of these are listed in Table 1. In each case the full liquefaction is confirmed in the free-field at the pile level. In all cases the co-seismic settlements are significantly larger than the post-seismic settlements.

Table 1. Excess pore pressure ratios and settlements of pile groups in different centrifuge tests

Pile Group ID	At pile tip level		Settlement (mm)	
	Depth (m)	Excess pore pressure ratio r_u	Co-seismic	Post-seismic
S1	15.2	1.00	1137	317
S2	15.2	0.96	635	14
S5	26.4	1.00	1480	88
S6	26.4	1.00	2342	144

DISCUSSION

In all the three examples considered in this paper i.e. SDOF structure, NRB model structure and the pile groups liquefaction induced settlements were considered in the co-seismic and post-seismic periods. In each case it was shown that the co-seismic settlements are much larger than the post-seismic settlements both in terms of the magnitude and rate of settlement. It is interesting to see in each of these cases the changes in the rate of settlement at the end of earthquake loading (see Figs. 5, 12 and 17). It has been shown from the excess pore pressure records that the soil below the respective structures remain at high pore pressure and does not start to dissipate immediately at the end of the earthquake loading. This raises an interesting question on why the rate of settlement seems to change at the end of the earthquake loading. A hypothesis on liquefied soil behaviour is proposed to answer this question.

Let us consider a loose, saturated, horizontal sand layer, for example the free-field in the SDOF structure centrifuge test BM1. In order to sustain large rate of settlement the pore fluid must be able to move quickly from the base of the model to the soil surface. This can only be achieved if the permeability of the liquefied sand is much greater perhaps by a factor 2 or 3 compared to permeability of the un-liquefied ground. This increased permeability seems to last only while the earthquake shaking is present.

In the presence of a structure on liquefied ground e.g. SDOF structure, NRB structure or a pile group, the rate of settlement is also influenced by the soil stiffness. Again large rates of settlement are possible if the soil's bulk modulus is decreased by perhaps an order of magnitude. The decrease in soil's bulk modulus is, in fact, linked to the permeability of the soil i.e. more easy it is to drain the pore fluid from liquefied ground the lower will be its bulk stiffness.

Schofield (1981) and more recently Muhunthun and Schofield (2000) have described the flow problems in liquefied soils. Schofield describes 'liquefaction phenomena such as boiling, piping etc occur when the stress path of soil suffering liquefaction reaches the Critical State Line in q-p' space. This will cause the soil to suffer fracture allowing gaps to open between soil grains and the permeability of the soil to increase many fold'. This historical perspective seems to agree with the hypothesis proposed earlier that the permeability of liquefied soil must increase and its bulk stiffness must decrease.

CONCLUSIONS

Settlement of structures following earthquake induced soil liquefaction is an important area of research. With increase in the popularity of Performance based Design, engineers will be required to estimate the settlement of structures accurately. This requires a better understanding of the failure mechanisms of structures founded on soil strata vulnerable to liquefaction.

In this paper three different boundary value problems are considered with liquefaction playing an important role in each case. The centrifuge test data that focuses on the settlement of the ground surface and the structures were considered along with the excess pore pressure data and some structural acceleration data. It was shown for all the three boundary value problems the co-seismic settlements were much larger than the post-seismic settlements. Further the rate of settlement was much higher during the co-seismic period compared to the post-seismic period.

A hypothesis on the behaviour of liquefied soil was then proposed to accommodate these changes in soil behaviour during and after the earthquake loading. It is proposed that the permeability of the liquefied soil must increase substantially to allow for the higher rate of settlement of level ground with no structures present. In the case where the structures are present on liquefiable ground the bulk modulus of the liquefied soil must decrease substantially again to accommodate for the increased rate of settlement.

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